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Using Nonlinear Modeling to Predict Peak Discharge and Average Breach Width from a Breached Embankment

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ABSTRACT

Some statistical models have been studied to predict peak discharge and average breach width from a breached embankment dam based on past dam failure data. The suggested models peak discharge Q_p, has related to average width of the embankment W_{avg}, water volume above the breach bottom at the time of failure V_w, height of water above the breach bottom at the time of failure H_w, height of the embankment above the breach bottom H_b. Our models are presented with different independent variables for peak outflow prediction. In addition, average breach width B_{avg}, has related to V_w and H_b besides failure mode for empirical model. A comparison between the estimated with actual data shows reasonable agreement to proposed models.

INTRODUCTION

Embankments are constructed for the retention of water because of irrigation and supply. Embankment Failure will cause the loss of life and property. The prediction of parameter breaches are important steps in managing the risks from potential embankment failure. Dam failure studies have been of great interest for many researchers. Many investigations have been developed methods that can be used to predict the peak discharge and dam breaching parameters through a breached embankment dam. Most of these relations use the reservoir volume and height, or some simple combination of these variables to conduct regression analysis. Predicting the breach peak outflow depends on the dam breaching process and can be conducted by identifying key breaching and reservoir parameters and relating them. Statistical modeling is an appropriate approach which is based on analysis of the datasets of numerous well-documented historical dam failures (SCS, 1981;USBR,1982; Hagen, 1982; MacDonald and Langridge-Monopolis, 1984; Singh and Snorrason, 1984; Costa, 1985; Evans, 1986; Froehlich, 1995; Webby, 1996; Walder and O'Connor, 1997; Wahl, 1998; Barker and Schaefer, 2007; Froehlich, 2008; Xu and Zhang, 2009; Pierce et al. 2010; Thornton et al. 2011; Hooshyaripor et al. 2014; De Lorenzo and Macchione, 2014; Azimi et al. 2015; Froehlich, 2016). Some researchers have focused on numerically studying (hydrodynamic models) the flood wave propagation due to dam breaks (Ponce et al. 2003; Tsai, 2005). However, numerical hydrodynamic models are not entirely reliable and can only give guidance for better understanding of the dam-break phenomenon (Carling et al. 2009).

In this study, several Statistical models are developed to predict peak discharge and average breach width from a breached embankment dam based on the evaluation of measured outflows from the datasets of historical dam failures gathered from the literature. Until now, various empirical equations have been developed using curve fitting techniques for estimating the peak outflow discharge and average breach width as a function of H_w and V_w. The empirical models peak discharge Q_p, has related to an average width of the embankment (between the toes of the downstream and upstream slopes) W_{avg}, water volume above the breach bottom at the time of failure V_w, height of water above the breach bottom at the time of failure H_w, height of the embankment above the breach

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bottom H_b. Models with different independent variables for peak outflow prediction are presented. And also for empirical model average breach width B_{avg} has related to V_w and H_b. The models have been tested by using historical failure database and results showing reasonable agreement for all.

MATERIALS AND METHODS

Embankment Dam Failure Data

Totally 1443 failure cases of earth dams in the database were utilized to study earth dam failures through a statistical analysis by Xu and Zhang (2009). To evaluate the parameters, needed to apply the empirical breach formation models, data from 41 embankment dam failures (e.g., dam height, estimated peak outflow, water-storage volume, embankment length, etc.) were assembled from a variety of sources and a summary of these embankment dam failures is presented in Table 1.

Breaching Parameters

A dam breach often has a trapezoidal shape, with the geometric parameters of breach depth H_b, breach top width B_t, average breach width B_{avg}, breach bottom width B_b, and breach side slope factor Z, as shown in Figure 1. Any combination of three of the five geometric parameters determines the breach shape and size.

Mode of failure

Typically, failure mode can be divided into two groups: Failure of overtopping and Piping. Overtopping failures usually begin with head cutting at the downstream toe and advance upstream until the erosion reaches the dam crest and reservoir surface. This type of failure may occur in any type of dams, but it is mostly dangerous for embankment dams because it washes away or erodes very quickly the dam's materials.

McCook (2004) defines piping as inter-granular seepage that occurs through a soil body which has no preferential flow paths. Piping is also sometimes referred to as backwards erosion piping because the erosion typically occurs from downstream to upstream.

Models for prediction Peak Breach Discharges and Dam Breaching Parameters

Empirical methods are used to predict average breach width, as well as to predict peak breach discharges. The empirical approach relies on statistical analysis of data obtained from documented failures. These methods have reasonably good correlation when comparing predicted values to actual observed values. The most basic statistical process generally involves plotting data for variables extracted from the dam failure dataset. Data from 41 embankment dam failures gathered from the literature are assembled to evaluate parameters needed in empirical models to predict peak discharge and average breach width from a breached embankment dam. Different independent variables are used for peak outflow prediction and three models are presented with four, three and two independent variables. As well, three models are estimated for empirical model average breach width with two independent variables (V_w and H_b) besides failure mode.

RESULTS AND DISCUSSIONS

Model with four independent variables

The height of water directly at the reservoir before breach, H_w, reservoir water volume at the time of failure, V_w, average width of the embankment, W_{avg}, and height of the embankment above the breach bottom, H_b, are used as input data (independent variables) and peak outflow through the breach, Q_p, is considered as the output (dependent variable). This study proposes a dimensionally homogenous model as

$$Q_P = \alpha_1 \sqrt{g H_w^5} \left(\frac{V_w^{\alpha_2} H_b^{\alpha_3} W_{avg}^{\alpha_4}}{H_w^{3\alpha_2+\alpha_3+\alpha_4}} \right) \tag{1}$$

in which the expression in the parentheses is a dimensionless term and radical term has a dimension as flow discharge, g is acceleration of gravity and regression coefficients α_1 to α_4 are determined via optimization procedure and using field data. Eq. (1), can be written as

$$Q_P = \alpha_1 \sqrt{g} V_w^{\alpha_2} H_b^{\alpha_3} W_{avg}^{\alpha_4} H_w^{2.5-(3\alpha_2+\alpha_3+\alpha_4)} \tag{2}$$

This equation with four regression parameters offers dimensional consistency. To determine the regression parameters, the root-mean squared error (*RMSE*) of Q_p is minimized as an objective function using the Solver toolbox of Microsoft Excel. Using the 41 case studies reported in Table 1, the regression coefficients were obtained. The final result is:

$$Q_P = 0.0118 \sqrt{g} V_w^{0.586} H_b^{0.150} W_{avg}^{-0.251} H_w^{0.843} \tag{3}$$

For assessing the performance of the proposed Eq. (3), two different criteria were used, the root-mean squared error (*RMSE*) and Nash-Sutcliffe criterion/efficiency coefficient (*E*). Applying Eq. (3), the values of RMSE and E, for the 41 case studies reported in Table 1 are respectively as 1981 and 0.984. Comparing powers of independent variables of Eq. (3), shows that the height of the embankment above the breach bottom H_b, has a minimum effect on the peak outflow through the breach, Q_p and may be neglected.

Model with three independent variables

Similarly by considering H_w, V_w and W_{avg} as independent variables and using the 41 case studies reported in Table 1 the following dimensionally homogenous model are obtained:

$$Q_P = 0.0122\sqrt{g}V_w^{0.580}W_{avg}^{-0.228}H_w^{0.988} \tag{4}$$

Applying Eq. (4), the values of *RMSE* and *E*, for the 41 case studies reported in Table 1 are respectively as 1985 and 0.984. As noted, H_b can be ignored from independent variables without any loss of efficiency. Additional independent variables should not be added to a predictive model unless the increased accuracy justifies the increase in model complexity. A pursue of Eq. (4), shows that the W_{avg}, has a minimum effect on the peak outflow Q_p and may be neglected.

Model with two independent variables

And finally by considering H_w and V_w as independent variables and using the 41 case studies reported in Table 1 the following dimensionally homogenous model is proposed for the peak outflow Q_p:

$$Q_P = 0.0094\sqrt{g}V_w^{0.581}H_w^{0.757} \tag{5}$$

The values of RMSE and E for Eq. (5) are respectively as 2084 and 0.982 as reported in Table 1. In Table 2, the performance of models developed in this study along with the models proposed by the other researchers is reported in the 41 case studies mentioned in Table 2 and in the 93 case studies reported in Azimi et al. (2015). Based on the Table 2, Eq. (3) proposed with four independent variables (H_w, V_w, H_b and W_{avg}) and without considering the mode of failure is preferable to the author model in terms of RMSE and E. It is interesting that Eqs. (4) and (5) proposed in this paper with only three and two independent variables, respectively, are also preferable to the complex model proposed which needs more input data. The model of Froehlich(2016) is preferable to his previous model developed in (1995) in terms of RMSE and E. However, the simple Eq. (5) proposed with only two independent variables is preferable to all other models in terms of *RMSE*, *E* and predictive power.

This model is developed using the 41 case studies, but such as the model proposed by Azimi et al. (2015) which is developed using the 70 case studies, is applicable to a wider range (93 case studies) as shown in Table 2.

A comparison of measured and predicted values of Q_p for different models reported in Table 2 is shown in Figure 2 for the 41 dams. Predictions given by Azimi et al. (2015), Froehlich (2016), and proposed three new models in this discussion provide generally good results. Though, the simple Eq. (5) proposed by the discussers with only two independent variables is preferable to all other models in terms of *RMSE*, *E* and simplicity.

A comparison of measured and predicted values of Q_p for the models which can be used in wider range (93 case studies) is shown in Figure 3. Eq. (5) of this study and predictive model proposed by Azimi et al. (2015) with only two independent variables are preferable to all other models in terms of RMSE, E.

Models with different mode failures for average breach width prediction

The breaching of embankment dams is an erosion process of the embankment material by flow of water either over or through the dam. The failure of earth dams is in a progressive mode, which mainly involves the erosion process over a period of time. The prediction model for average breach width was found from multiple linear regression analysis by Froehlich (2008). The average breach width (B_{avg}) was calculated as:

$$B_{avg} = 0.27K_oV_W^{0.32}H_b^{0.4} \tag{6}$$

Where K_o = Failure Mode Factor; H_b = Height of breach; V_w = Reservoir volume stored. The values are 1.0 and 1.3 for piping and overtopping failures, respectively.

Three nonlinear models are developed for prediction of average breach width based on failure mode and with two independent variables (V_w and H_b). A dimensionally homogenous model is obtained as follows:

$$B_{avg} = \alpha_1 V_W^{\alpha_2} H_b^{1-3\alpha_2} \tag{7}$$

Where regression coefficients (α_1 *and* α_2) are determined via optimization procedure and using field data.

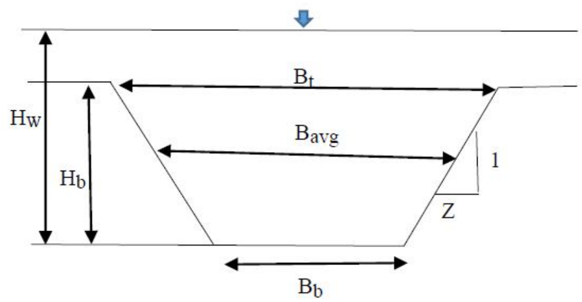

Figure 1. Geometric parameters of an idealized dam breach

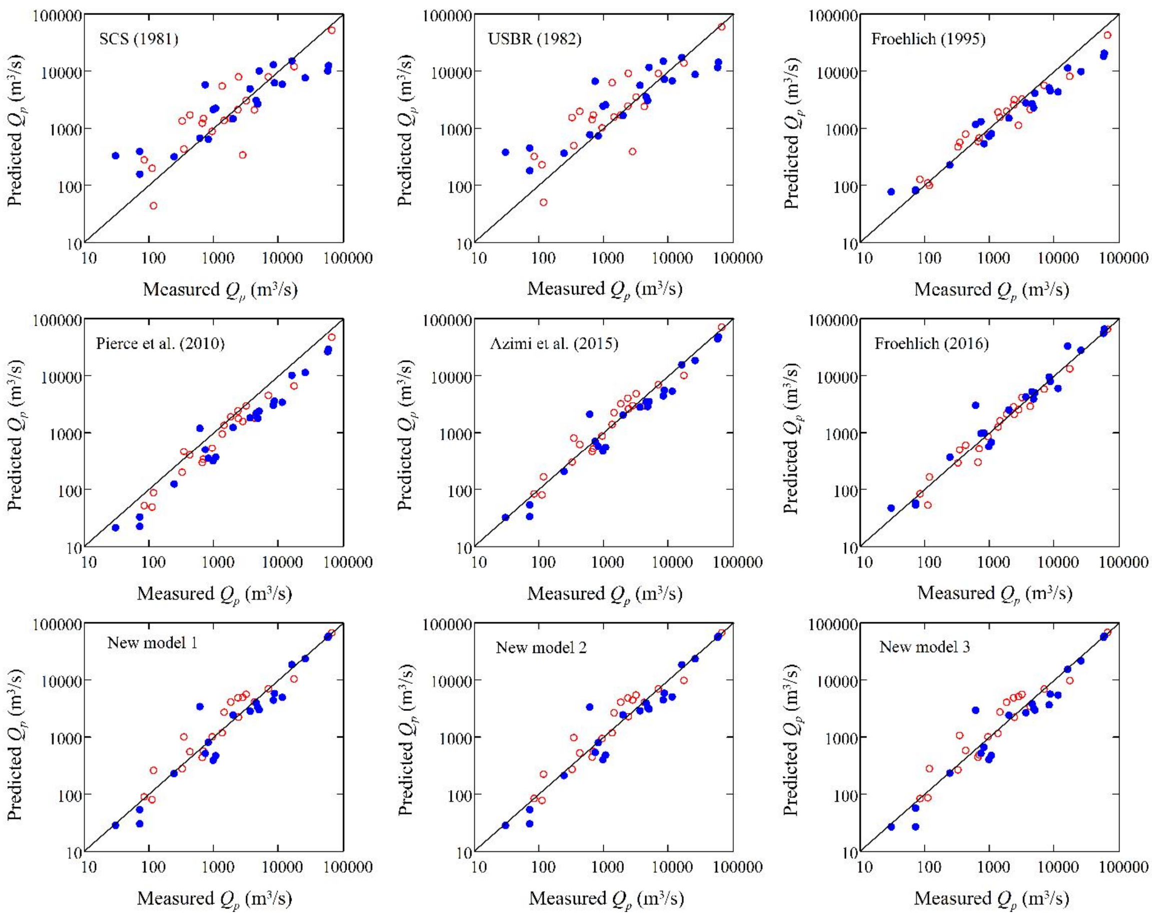

Figure 2. Comparison of measured and predicted values of Qp for different models (41 case studies).

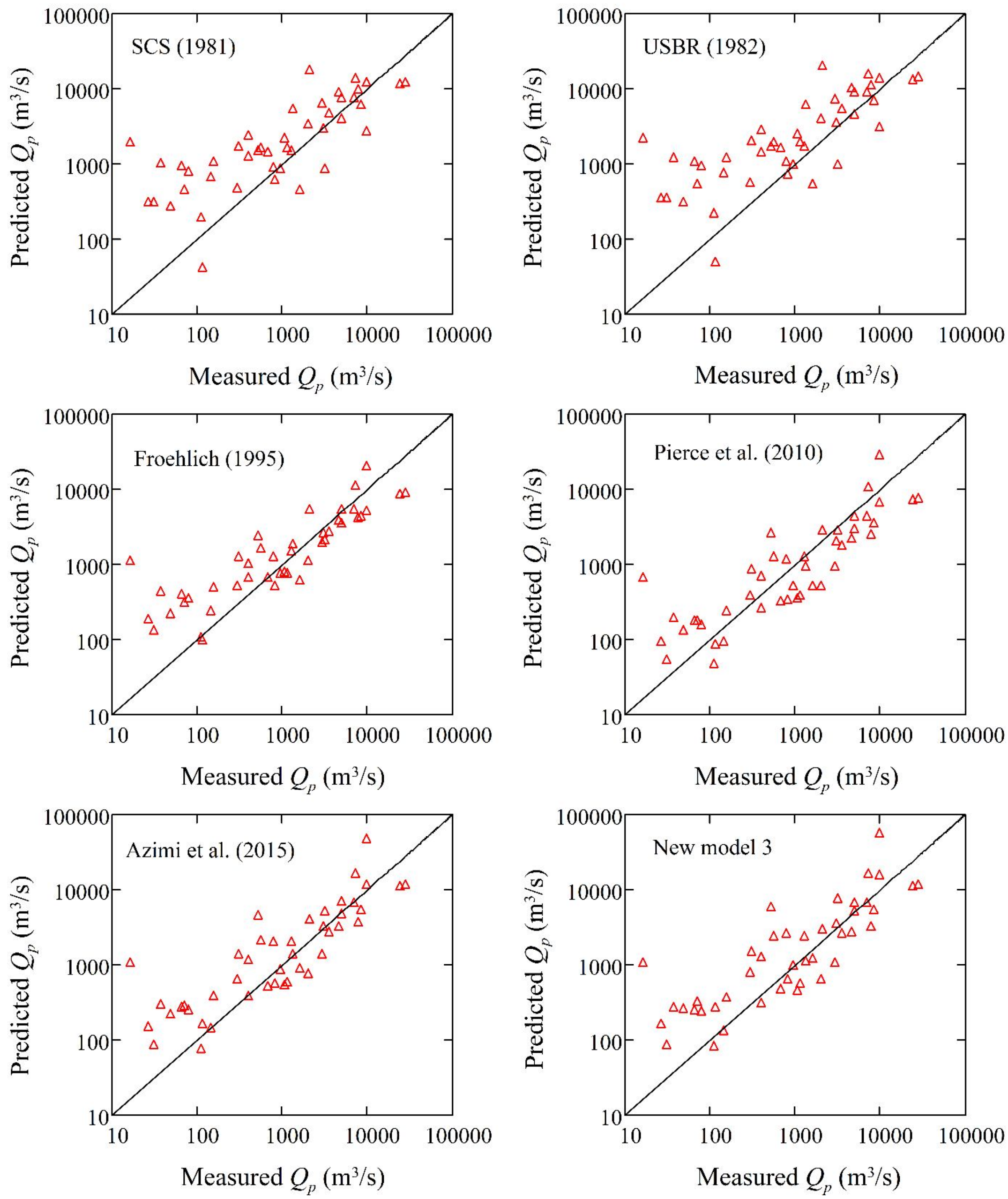

Figure 3. Comparison of measured and predicted values of Qp for different models (93 case studies).

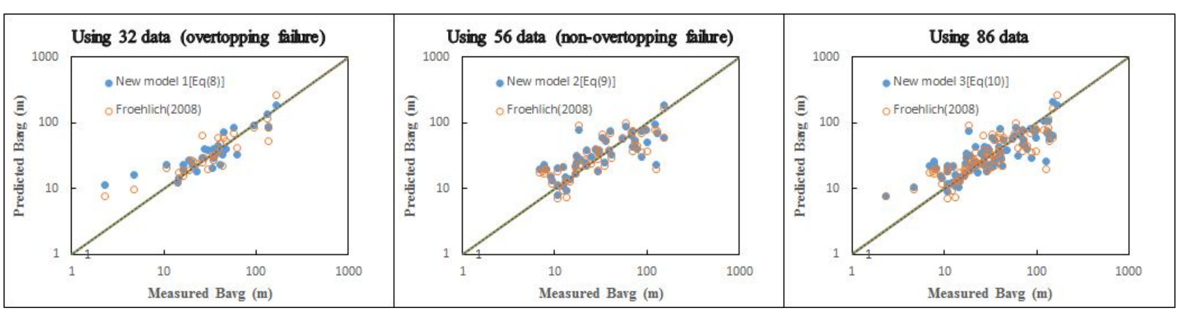

Figure 4. Comparison of measured and predicted values of Bavg.

Table 1. Original datasets used for developing equation for peak outflow prediction

Number	Dam name	Location	Failure Mode	$W_{avg}(m)$	$V_w(Mm^3)$	$H_w(m)$	$H_b(m)$	$Q_p(m^3/s)$
1	Apishapa, Colorado	USA	P	82.4	22.8	28	31.1	6,850
2	Baldwin Hills,California	USA	P	59.6	0.95	12.2	21.3	420
3	Banqiao, Henan,Province	China	O	97	701	31.9	29.5	56,300
4	Bass Haven Lake, Texas	USA	O	22.9	0.641	4.9	9.2	240
5	Belci, Bacău County	Romania	O	37.8	12.7	15.5	15	4,700
6	Big Bay Lake,Mississippi	USA	P	20.4	17.5	13.6	14	4,160
7	Bila Desna	Czech Republic	P	29.6	0.29	10.7	14.6	320
8	Bilberry	U.K.	O	62.5	0.327	23.6	23	725
9	Bradfield (Dale Dyke)	U.K.	P	76	3.2	28	29	2,370
10	Butler Valley,Arizona	USA	O	9.63	2.38	7.16	7.16	810
11	Castlewood, Colorado	USA	O	47.4	6.17	21.6	21.3	3,570
12	Centralia (Seminary Hill) Reservoir No. 3, Washington	USA	O	10.1	0.01333	5.5	6.1	71
13	Delhi, Iowa	USA	O	31.5	12.2	11.2	11	1,950
14	FP&L Martin Plant, Florida	USA	P	27.7	125	5.09	9.14	2,750
15	Fred Burr, Montana	USA	P	30.8	0.75	10.2	10.2	654
16	French Landing, Michigan Frenchman Creek,	USA	P	34.3	3.87	8.53	14.2	929
17	Frenchman Creek,Montana	USA	P	37.3	16	10.8	12.5	1,420
18	Hästberga	Sweden	O	12.7	30	7.35	7	600
19	Hatchtown, Utah	USA	O	44.8	16	16.8	18.3	4,440
20	Hell Hole, California	USA	P	103	30.6	35.1	56.4	17,000
21	Ireland Reservoir No. 5, Colorado	USA	P	18	0.16	3.81	5.18	110
22	Kelly Barnes, Georgia	USA	P	19.4	0.777	11.3	12.8	680
23	Lake Avalon, New Mexico	USA	P	42.7	31.5	13.7	14.6	2,320
24	Laurel Run,Pennsylvania	USA	O	40.5	0.555	14.1	13.7	1,050
25	Lily Lake, Colorado	USA	O	13.2	0.0925	3.35	3.66	71
26	Little Deer Creek,Utah	USA	P	63.1	1.36	22.9	27.1	1,330
27	Lower Latham,Colorado	USA	P	25.7	7.08	5.79	7.01	340
28	Lower Otay,California	USA	O	53.3	56.9	39.6	39.6	15,800
29	Lower Two Medicine, Montana Oro´ s	Brazil	P	33.3	29.6	11.3	11.3	1,800
30	Or´os	Brazil	O	110	660	35.8	35.5	58,000
31	Porter Hill, Oregon	USA	O	12	0.015	5	5.8	30
32	Prospect, Colorado	USA	P	13.1	3.54	1.68	4.42	116
33	Puddingstone,California	USA	O	43	0.432	13.7	13.7	960

Table 1. Continue

34	*Quail Creek, Utah*	*USA*	*P*	*56.6*	*30.8*	*16.7*	*21.3*	*3,110*
35	*Rito Manzanares, New Mexico*	*USA*	*P*	*13.4*	*0.12*	*4.57*	*7.32*	*83*
36	*Schaeffer, Colorado*	*USA*	*O*	*80.8*	*4.44*	*31.9*	*30.5*	*4,930*
37	*Shimantan, Henan Province*	*China*	*O*	*58*	*167*	*27.4*	*25.8*	*25,300*
38	*South Fork, Pennsylvania*	*USA*	*O*	*64*	*18.9*	*24.6*	*24.4*	*8,500*
39	*Taum Sauk Reservoir, Missouri*	*USA*	*O*	*46.8*	*5.39*	*36.5*	*36.3*	*8,180*
40	*Teton, Idaho*	*USA*	*P*	*250*	*310*	*77.4*	*86.9*	*65,120*
41	*Zhuguo, Henan Province*	*China*	*O*	*99*	*18.5*	*23.8*	*23.5*	*11,200*

Table 2. Proposed empirical equations for peak outflow prediction by different researchers

Investigators	Number of case study used in model development	Proposed empirical model*	Applied to 41 case studies reported in Froehlich (2016)		Applied to 93 case studies reported in Azimi et al. (2015)	
			RMSE	E	RMSE	E
SCS (1981)	13	$Q_P = 16.6 H_W^{1.85}$	10947	0.505	8408	0.475
USBR (1982)	21	$Q_P = 19.1 H_W^{1.85}$	10446	0.549	8239	0.496
Froehlich (1995)	22	$Q_P = 0.607 H_W^{1.24} V_W^{0.295}$	9633	0.616	8011	0.523
Pierce et al. (2010)	87	$Q_P = 0.038 H_W^{1.09} V_W^{0.475}$	7891	0.743	7635	0.567
Azimi et al. (2015)	70	$Q_P = 0.0166 g^{0.5} V_W^{0.5} H_W$	**3359**	**0.953**	**6633**	**0.673**
Froehlich (2016)	41	$Q_P = 0.0175 \times K_M \times K_H \times g^{0.5} H_W^{0.5} V_W^{0.5} H_b W_{ave}^{-0.5}$ $k_M = \begin{cases} 1 & \text{for non} - \text{overtopping fa} \\ 1.85 & \text{for overtopping fa} \end{cases}$ $k_H = \begin{cases} 1 & \text{for } H_b \leq H_s \\ \left(\frac{H_b}{H_s}\right)^{1/8} & \text{for } H_b > H_s \end{cases}$ $H_s = 6.1\ \text{m}$	3199	0.958	**	**
New model 1 [Eq. (3)]	41	$Q_P = 0.0118 g^{0.5} H_W^{0.843} V_W^{0.586} H_b^{0.150} W_($	1981	0.984	**	**
New model 2 [Eq. (4)]	41	$Q_P = 0.0122 g^{0.5} H_W^{0.988} V_W^{0.580} W_{ave}^{-0.228}$	1985	0.984	**	**
New model 3 [Eq. (5)]	41	$Q_P = 0.0094 g^{0.5} H_W^{0.757} V_W^{0.581}$	**2084**	**0.982**	**6866**	**0.650**

*All models are in SI units: Q_p= m^3/s, g=9.81 m/s^2, V_w=m^3, H_w = m, and H_b=m, W_{avg}=m and H_s = 6.1 m.

**Not applicable to all case studies because of unavailable information for H_b and W_{avg} in some cases.

Table 3. Original datasets used for developing equation for average breach width prediction

Number	*Dam name*	*Location*	*Failure mode**	*Vw(Mm^3)*	*Hw(m)*	*Hb(m)*	*Bavg(m)*
1	*Buckhaven No. 2, Tennessee*	*USA*	*O*	*0.0247*	*6.1*	*6.1*	*4.72*
2	*Butler, Arizona*	*USA*	*O*	*2.38*	*7.16*	*7.16*	*62.5*
3	*Castlewood*	*USA*	*O*	*6.17*	*21.6*	*21.3*	*44.2*
4	*Clearwater Lake Dam, Georgia*	*USA*	*O*	*0.466*	*4.05*	*3.78*	*22.8*
5	*Coedty*	*U.K.*	*O*	*0.311*	*11*	*11*	*42.7*
6	*Danghe*	*China*	*O*	*10.7*	*24.5*	*25*	*58*
7	*Elk City*	*USA*	*O*	*1.18*	*9.44*	*9.14*	*36.6*
8	*Erlangmiao*	*China*	*O*	*0.196*	*9*	*9*	*18.8*
9	*Fengzhuang*	*China*	*O*	*0.625*	*8*	*8*	*35*
10	*Goose Creek, S. Carolina*	*USA*	*O*	*10.6*	*1.37*	*4.1*	*26.4*
11	*Grand Rapids*	*USA*	*O*	*0.255*	*6.4*	*6.4*	*10.7*
12	*Huoshishan*	*China*	*O*	*0.22*	*16*	*16*	*30*
13	*Hutchinson Lake Dam, Georgia*	*USA*	*O*	*1.17*	*4.42*	*3.75*	*33.4*
14	*Johnstown*	*USA*	*O*	*18.9*	*24.6*	*24.4*	*94.5*
15	*Kelly Barnes*	*USA*	*O*	*0.777*	*11.3*	*12.8*	*27.3*
16	*Kraftsmen's Lake Dam, Georgia*	*USA*	*O*	*0.177*	*3.66*	*3.2*	*14.5*
17	*Lake Philema Dam, Georgia*	*USA*	*O*	*4.78*	*9*	*8.53*	*47.2*
18	*Laurel Run, Penn.*	*USA*	*O*	*0.555*	*14.1*	*13.7*	*35.1*
19	*Lower Otay*	*USA*	*O*	*49.3*	*39.6*	*39.6*	*133*
20	*Merimac (Upper) Lake Dam, Georgia*	*USA*	*O*	*0.0696*	*3.44*	*3.05*	*14.2*
21	*Mossy Lake Dam, Georgia*	*USA*	*O*	*4.13*	*4.41*	*3.44*	*41.5*
22	*Oros*	*Brazil*	*O*	*660*	*35.8*	*35.5*	*165*
23	*Potato Hill*	*USA*	*O*	*0.105*	*7.77*	*7.77*	*16.5*
24	*Rainbow Lake, Mich.*	*USA*	*O*	*6.78*	*10*	*9.54*	*38.9*
25	*Renegade Resort Lake, Tenn.*	*USA*	*O*	*0.0139*	*3.66*	*3.66*	*2.29*
26	*Schaeffer Reservoir*	*USA*	*O*	*4.44*	*30.5*	*30.5*	*137*
27	*Statham Lake*	*USA*	*O*	*0.564*	*5.55*	*5.12*	*21*
28	*Trout Lake, N.C.*	*USA*	*O*	*0.493*	*8.53*	*8.53*	*26.2*
29	*Upper Pond*	*USA*	*O*	*0.222*	*5.18*	*5.18*	*16.5*
30	*Wanshangang*	*China*	*O*	*1.5*	*12*	*12*	*40*
31	*Winston*	*USA*	*O*	*0.662*	*6.4*	*6.1*	*19.8*
32	*Zhugou*	*China*	*O*	*18.43*	*23.5*	*23.5*	*135*
33	*Apishapa*	*USA*	*P*	*22.2*	*28*	*31.1*	*86.5*
34	*Baldwin Hills, Calif.*	*USA*	*P*	*0.91*	*12.2*	*21.3*	*25*

Table 3.Continue

35	Bayi	China	P	23	28	30	40
36	Bullock Draw Dike, Utah	USA	P	0.74	3.05	5.79	12.5
37	Davis Reservior	USA	P	58	11.58	11.9	18.3
38	East Fork Pond River, Kentucky	USA	P	1.87	9.8	11.4	17.2
39	Emery, Calif.	USA	P	0.425	6.55	8.23	10.8
40	Fogelman, Tennessee	USA	P	0.493	11.1	12.6	7.62
41	Frankfurt	Germany	P	0.352	8.23	9.75	6.9
42	French Landing	USA	P	3.87	8.53	14.2	27.4
43	Frenchman Creek, Mont.	USA	P	16	10.8	12.5	54.6
44	Gouhou	China	P	3.18	44	48	99.5
45	Haas Pond, Connecticut	USA	P	0.0234	2.99	3.96	10.7
46	Hart	USA	P	6.35	10.7	10.8	73.9
47	Hell Hole	USA	P	30.6	35.1	56.4	121
48	Horse Creek	USA	P	12.8	7.01	12.8	73.1
49	Huqitang	China	P	0.424	5.1	9	7.5
50	Iowa Beef Processors, Washington	USA	P	0.333	4.42	4.57	16.8
51	Ireland No. 5, Colo.	USA	P	0.16	3.81	5.18	13.5
52	Jacobs Creek, Penn.	USA	P	0.423	20.1	21.3	17.5
53	Johnston City, Ill.	USA	P	0.575	3.05	5.18	8.23
54	La Fruta, Tex.	USA	P	78.9	7.9	14	58.8
55	Lake Avalon, N.M.	USA	P	31.5	13.7	14.6	130
56	Lake Frances	USA	P	0.789	14	17.1	18.9
57	Lake Genevieve, Kentucky	USA	P	0.68	6.71	7.92	16.8
58	Lake Latonka	USA	P	4.09	6.25	8.69	39.2
59	Lambert Lake, Tenn.	USA	P	0.296	12.8	14.3	7.62
60	Lawn Lake, Colo.	USA	P	0.798	6.71	7.62	22.2
61	Lily Lake, Colo.	USA	P	0.0925	3.35	3.66	10.8
62	Little Deer Creek	USA	P	1.36	22.9	27.1	29.6
63	Long Branch Canyon, Calif.	USA	P	0.284	3.17	3.66	9.14
64	Lower Latham, Colo.	USA	P	7.08	5.79	7.01	79.2
65	Lower Two Medicine	USA	P	29.6	11.3	11.3	67
66	Lyman	USA	P	35.8	16.2	19.8	97
67	Lynde Brook	USA	P	2.88	11.6	12.5	30.5
68	Melville, Utah	USA	P	24.7	7.92	9.75	32.8
69	Niujiaoyu	China	P	0.144	7.2	7.2	13
70	Otter Lake	USA	P	0.109	5	6.1	9.3

Table 3. Continue

71	*Pierce Reservoir, Wyo.*	*USA*	*P*	*4.07*	*8.08*	*8.69*	*30.5*
72	*Prospect*	*USA*	*P*	*3.54*	*1.68*	*4.42*	*88.4*
73	*Quail Creek*	*USA*	*P*	*30.8*	*16.7*	*21.3*	*70*
74	*Rito Manzanares*	*USA*	*P*	*0.0247*	*4.57*	*7.32*	*13.3*
75	*Shanhu*	*China*	*P*	*1.78*	*12.5*	*13*	*41*
76	*Sheep Creek*	*USA*	*P*	*2.91*	*14.02*	*17.1*	*22*
77	*Spring Lake*	*USA*	*P*	*0.136*	*5.49*	*5.49*	*14.5*
78	*Teton*	*USA*	*P*	*310*	*77.4*	*86.9*	*151*
79	*Trial Lake*	*USA*	*P*	*1.48*	*5.18*	*5.18*	*21*
80	*Wheatland No. 1, Wyo.*	*USA*	*P*	*11.6*	*12.2*	*13.7*	*35.4*
81	*Wilkinson*	*USA*	*P*	*0.533*	*3.57*	*3.72*	*29*
82	*Hatchtown, Utah*	*USA*	*P*	*14.8*	*16.8*	*18.3*	*151*
83	*Caulk Lake, Kentucky*	*USA*	*P*	*0.698*	*11.1*	*12.2*	*35.1*
84	*Buffalo Creek, W. Va.*	*USA*	*P*	*0.483525*	*14.02*	*14*	*125*
85	*Sinker Creek*	*USA*	*P*	*3.330401*	*21.34*	*21.3*	*70.6*
86	*Bearwallow Lake, N.C.*	*USA*	*P*	*0.0493*	*5.79*	*6.4*	*12.2*

* O=overtopping; P=piping

Table 4. Proposed empirical equations average breach width prediction by different researchers

New models	*RMSE*	*E*	*Model of Froehlich 2008*	*RMSE*	*E*
$B_{avg} = 1.45 V_W^{0.13} H_b^{0.61}$	*16.88*	*0.831*	$B_{avg} = 0.27 k_o V_W^{0.32} H_b^{0.04}$	*27.60*	*0.55*
$B_{avg} = 0.45 V_W^{0.26} H_b^{0.22}$	*28.33*	*0.48*	$k_o = \begin{cases} 1.3 & \text{for overtopping failures} \\ 1.0 & \text{for other failuers} \end{cases}$	*29.39*	*0.44*
$B_{avg} = 0.58 V_W^{0.23} H_b^{0.31}$	*25.97*	*0.58*		*28.72*	*0.48*

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Model for overtopping failure

The reservoir water volume at the time of failure, V_w, and height of the embankment above the breach bottom H_b, are used as input data (independent variables) and average breach width, B_{avg}, is considered as the output (dependent variable). To determine the regression parameters (α_1 *and* α_2) for the cases of overtopping failures, the root-mean squared error (RMSE) of B_{avg}, is minimized as an objective function using the Solver toolbox of Microsoft Excel. Using the 32 case studies reported in Table 3, the regression coefficients were obtained. The final result is:

$$B_{avg} = 1.45 V_W^{0.13} H_b^{0.61} \qquad (8)$$

Two different criteria (*RMSE* and *E*) were used for assessing the performance of the proposed Eq. (8). Applying Eq. (8), the values of *RMSE* and *E*, for the 32 case studies reported in Table 4 are respectively as 16.88 and 0.831.

Model for non-overtopping failure

Using the same analysis procedure, model for the case of a non-overtopping failure is expressed as and by considering H_b and V_w as independent variables and using the 56 case studies reported in Table 3 the following dimensionally homogenous model is proposed for the average breach width B_{avg}:

$$B_{avg} = 0.45 V_W^{0.26} H_b^{0.22} \tag{9}$$

Applying Eq. (9), the values of *RMSE* and *E*, for the 56 case studies reported in Table 3 are respectively as 28.33 and 0.480.

Model for overtopping and non-overtopping failure

Regardless of the type of dam failure and using all data (86 case studies) reported in Table 3, the regression coefficients are obtained as:

$$B_{avg} = 0.58 V_W^{0.23} H_b^{0.31} \tag{10}$$

The values of *RMSE* and *E* for Eq. (10) are 25.97 and 0.580respectively. In Table 4, the performance of models developed to predict the average breach width in this study along with the model proposed by Froehlich (2008) is reported. Based on the Table 4, the models of this study are preferable to model proposed by Froehlich (2008) in terms of *RMSE* and *E*. Figure 4 is shown a comparison of measured and predicted values of B_{avg} for the models of this study and predictive model proposed by Froehlich (2008). The Figure 4 shows that the Eq. ((8)-(10)) results with real data are a better fit than the model proposed by Froehlich (2008).

CONCLUSIONS

The embankments are massive structure, which impounds a large volume of water in the upstream of the reservoir. Embankment failure is disastrous to human life as well as property, hence, dam break analysis has abundant importance. In this study several nonlinear models are developed to predict peak discharge and average breach width from a breached embankment dam with applying different independent variables and finally historical data are used for testing the models. Four, three and two independent variables are used for predicting peak discharge. Comparing the power of different parameters of regression based equations, which were derived based on historical datasets, affirming that H_w and V_w value have more effect on the peak outflow discharge than H_b and W_{avg}. The simple model proposed predicting peak discharge with only two independent variables is preferable to the complex models proposed which need more input data and is applicable to a wider range of case studies. Moreover, three nonlinear models are presented for the prediction of average breach width based on failure mode and with two independent variables (V_w and H_b). Results show that the models of this study are preferable to model proposed by Froehlich (2008) in terms of *RMSE* and *E*.

REFERENCES

Azimi R. Vatankhah A.R. Kouchakzadeh S. (2015) Predicting peak discharge from breached embankment dams. E-Proc., 36th IAHR World Congress, Hague, Netherlands.

Barker B. Schaefer M. (2007) Dam safety guidelines-Technical Note 1: Dam break inundation analysis and downstream hazard classification. Washington State Dept. of Ecology, Water Resources Program, Dam Safety Office, Olympia, WA.

Carling P. Villanueva I. Herget J. Wright N. Borodavko P. Morvan H. (2009) Unsteady 1D and 2D hydraulic models with ice dam break for Quaternary megaflood. Altai Mountains, southern Siberia. Global and Planetary Change 70 (4): 24-34.

Costa J.E. (1985) Flood from dam failures. U.S. Geological Survey, Cascades volcano observatory, Vancouver, Washington, pp 439–463.

De Lorenzo G. Macchione F. (2014) Formulas for the peak discharge from breached earthfill dams. Journal of Hydraulic Engineering, 10.1061/(ASCE)HY.1943-7900.0000796, 56–67.

Evans S.G. (1986) The maximum discharge of outburst floods caused by the breaching of man-made and natural dams. Canadian Geotechnical Journal, 23(3):385–387

Froehlich D.C. (1995) Peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, 10.1061-(ASCE)0733-9496(1995)121:1(90): 90–97.

Froehlich D.C. (2008) Embankment dam breach parameters and their uncertainties. Journal of Hydraulic Engineering, 10.1061-(ASCE)0733-9429(2008) 134:12(1708), 1708–1721.

Froehlich D.C. (2016) Predicting peak discharge from gradually breached embankment dam. Journal of Hydrologic Engineering, 21(11): 04016041.

Hagen V. K. (1982) Re-evaluation of design floods and dam safety. Proc., 14th Congress of Int. Commission on Large Dams, Int. Commission on Large Dams, Paris, 475–491.

Hooshyaripor F. Tahershamsi A. Golian S. (2014) Application of copula method and nueral networks for predicting peak outflow from breached embankments. Journal of Hydro-environment Research, 8(3): 292–303.

MacDonald T.C. Langridge-Monopolis J. (1984) Breaching characteristics of dam failures. Journal of Hydraulic Engineering, 110:567–586

McCook D. K. (2004) A Comprehensive Discussion of Piping and Internal Erosion Failure

Mechanisms. 21th Annual Conf. of the Association of State Dam Safety Officials, ASDSO, Lexington, KY

Nash J. E. Sutcliffe J.V. (1970) River flow forecasting through conceptual models. Part I - A discussion of principles, Journal of Hydrology, 10: 282–290.

Pierce M.W. Thornton C.I. Abt S.R. (2010) Predicting peak outflow from breached embankment dams. Journal of Hydraulic Engineering, 15 (5): 338-349.

Ponce V.M. Taher-Shamsi A. Shetty A.V. (2003) Dam-breach flood wave propagation using dimensionless parameters. Journal of Hydraulic Engineering, 129 (10): 777-782.

Singh K.P. Snorrason A. (1982) Sensitivity of outflow peaks and flood stages to the selection of dam breach parameters and simulation models. State Water Survey Div. Report 289, Surface Water Section at Univ. of Illinois, Urbana, 111.

Soil Conservation Service (1981) Simplified dam-breach routing procedure. U.S. Dept. of Agriculture, Washington, DC.

Thornton C.I. Pierce M.W. Abt S.R. (2011). Enhanced predictions for peak outflow from breached embankment dams. Journal of Hydraulic Engineering, 10.1061-(ASCE)HE.1943-5584.0000288, 81–88.

Tsai C.W. (2005) Flood routing in mild-sloped rivers-wave characteristics and downstream backwater effect. Journal of Hydrology, 308 (4): 151-167.

United States Bureau of Reclamation (1982) Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams. Reclamation Planning Instruction No 82-11. US Department of the Interior, Bureau of Reclamation, Denver, p. 25.

Wahl T.L. (1998) Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment. Bureau of Reclamation, U.S. Department of the Inte rior, Denver, 60, Rep. no. DSO–98–004, 1998.

Walder J.S. O'Connor J.E. (1997) Methods for predicting peak discharge of floods caused by failure of natural and constructed earth dams. Water Resources Research, 33 (10), 12.

Webby M.G. (1996) Discussion of peak outflow from breached embankment dam. Journal of Water Resources Planning and Management, 122(4):316–317

Xu Y. Zhang L.M. (2009) Breaching parameters for earth and rockfill dams. Journal of Geotechnical and Geoenvironmental Engineering, 135(12):1957–1970.